20. Risk Analysis for Concrete Gravity Structures

Key Concepts

Within the context of this section, massive concrete spillways or other gravity-type concrete water retention structures are also referred to as concrete gravity dams. Historically, the leading cause of concrete gravity dam failures (for those founded on rock) has been related to sliding on planes of weakness within the foundation, most typically weak clay or shale layers within sedimentary rock formations. A few failures have also occurred along weak lift joints within masonry (and buttress) dams. This section focuses on risks associated with sliding instability of concrete gravity dams.

For concrete gravity dams founded on alluvial soils, the leading cause of failure is piping or "blowout" of the soil material from beneath the dam. Therefore, the reader is referred to the section on Internal Erosion and Piping Risks for Embankments for evaluating this potential failure mode, considering "backward erosion piping" of the foundation soils.

The heel of the dam is a location of sharp geometry change and as such is a point of singularity and stress concentration. Thus, the dam-foundation contact is typically the focus of most of the stability analysis. However, this typically is not the weak link in the dam-foundation system, unless the dam is founded on the foot wall of smooth discontinuity surfaces such as faults or bedding planes. The rough surface that results from blasting the dam keyway excavation typically provides a significant roughness or "dilation" component to the shear strength on this surface, which should be taken into account to the extent possible based on construction photographs and other information. If the surface clean-up is good, significant cohesion and tensile strength can result (as with lift joints).

When surface cleanup of lift joints is not good, weaker horizontal planes may occur within the dam body. For gravity dams constructed in blocks, the weaker planes may not "line up" across contraction joints, and if the joints are constructed with keys, considerable stability can result from load transfer to adjacent monoliths. This should be considered when evaluating the risks.

A line of functioning drainage holes in the foundation or dam body adds significantly to the sliding stability of concrete gravity dams by reducing water pressures (typically referred to as "uplift") along potential sliding surfaces. A decrease in water pressures increases the effective normal stress and frictional resistance. Research shows that drains remain effective even if a crack or open surface extends downstream of the drainage curtain as noted in nonlinear analysis guidelines (Mills-Bria et al, 2006), based on the Electric Power Research Institute (EPRI) research results (Amadei et al, 1991). However, drainage systems can become plugged over time if they are not maintained, and the drainage curtain can be offset under significant seismic displacements, thus becoming less effective.

Shear keys constructed within the contraction joints separating concrete monoliths are beneficial in that they can facilitate load transfer between monoliths. This could be important if one monolith or series of monoliths contains an unbonded lift joint or weak foundation conditions, whereby load in excess of the weak monolith(s) capacity could be transferred to adjacent stronger monoliths. Not all gravity dams contain shear keys within the contraction joints, but many do.

When a potential sliding plane is formed by a partially bonded and partially unbonded surface, care must be taken in assigning the shear strength to each portion. That is because the peak shear strengths may not be mobilized at compatible displacements. It may take much less shear displacement to mobilize the shear strength of a bonded joint than an unbonded joint, in which case it may not be appropriate to simply add the peak strengths determined from testing. Test results could be examined and new strength curves developed at compatible displacements.

A special discussion on tensile strength of concrete and the so called "cracked base" analysis is provided here, as estimating risks requires a somewhat different approach than that currently provided in design criteria documents. As opposed to designing a new dam, where conservative assumptions and criteria are appropriate to ensure that the dam does not slide for the design loads, estimating risks for an existing dam requires attempting to establish the most likely behavior.

Tensile Strength of Concrete

The tensile strength of concrete has been somewhat controversial over the years. Jerry Raphael published an often cited paper on the subject (Raphael, 1984). His basic conclusions were that:

- Direct tension tests are unreliable, and can be in error by as much as 50 percent (attributed to moisture gradients during drying which caused surface micro-cracking and an effective reduction on cross-sectional area in these types of tests – Cannon (1995) noted that the drilling process can also induce strains capable of causing micro-cracking of the core surface).
- Splitting tension tests are the most reliable means of determining tensile strength (potential zone of micro-cracking is loaded in compression).
- Tensile strength determined by static testing (from splitting tension tests) should be increased by 50 percent when used with seismic loadings, based on rapid loading tests where the samples are taken to failure in a fraction of a second representing one load cycle during an earthquake.

Raphael supported the fact that splitting tensile tests best represent tensile strength by converting strengths from modulus of rupture (flexural) tests results to tensile strength based on an evaluation of the stress and strain distribution in the samples, which shows the tensile strength should be about ¾ the modulus of rupture value. The results, suggest that both splitting tension tests and modulus of rupture tests produce a consistent pattern.

Raphael indicated that an apparent tensile strength can be used when performing linear elastic analyses to account for the non-linear strain that occurs prior to failure. The apparent strength is estimated as the failure tensile strain multiplied by the Modulus of Elasticity used in the analysis. It can be determined directly from modulus of rupture (flexural) tests, or can be estimated from equations he provides (which result in approximately a factor of 1.35 applied to the static strengths). However, this typically has not been used in practice due to discomfort with using the high strengths it produces, particularly for static loading. However, for dynamic loading, it can be considered, especially if it is taken that only one spike in the stress time history is sufficient to crack the concrete (considering that it represents a rapid load cycle similar to that experienced under dynamic testing).

Slightly over a decade later, Bob Cannon published additional information in a Corps of Engineers Engineering Pamphlet (Cannon, 1995). He examined direct tension and splitting tension test results for a variety of conventional concretes, and concluded that:

- Splitting tensile strengths can be used as a starting point. If these strengths are not available, then: (1) for compressive strengths less than 3,000 lb/in^2 the tensile strength is expected to vary between 10 and 15 percent of the compressive strength, or (2) for compressive strengths greater than 3,000 lb/in^2, the equations $1.7(f_c')^{2/3}$ (Raphael's equation) or $7(f_c')^{1/2}$, which are based on splitting test relationships, can be used to estimate tensile strength.
- The tensile strength values should be reduced by 10 percent if the maximum size aggregate is larger than 1½ inches (based on 6-inch x 12-inch cylinders).
- The strengths should be reduced by an additional 20 percent to adjust for direct tensile strength (particularly for examining vertical or "cantilever" stresses).

Cannon also gave recommendations for roller-compacted concrete (RCC) indicating that:

- Parent material tensile strength should be no higher than about 75 percent of the splitting tensile strength value, reduced by 10 percent if based on wet-screening of aggregates larger than 1½ inches.
- Joint tensile strength is similar to conventional concrete when properly cleaned, cured, and covered with a suitable mortar or bedding mix.

Cannon supported Raphael's conclusions that:

- The dynamic tensile strength of concrete is about 1.5 times the static tensile strength.
- For linear elastic finite element analysis, the apparent tensile strength of concrete is about 1.35 times the tested strength.

The intent of reviewing these important pieces of work is not to dictate the tensile stress parameters that should be used in a risk analysis. These need to be determined on a case-by-case basis using available information. However, the work by Cannon is important and often overlooked in estimating tensile strength. It is always preferred to have tested material properties from extracted core from the dam. The direct and splitting tensile strengths can significantly vary from dam to dam at shown in the Non-Linear Practices Manual (Mills-Bria et al, 2006).

Lift line strength is not only a function of the concrete strength but is greatly influenced by construction methods. Experience suggests that tensile strength across lift joints for modern concrete construction with good joint clean-up averages about 85 percent of the parent concrete strength. Good cleanup usually involves water curing the top of new concrete lifts, then "green cutting" or water blasting (sometimes sandblasting) the laitance from the top of a lift prior to placing the overlying concrete. Sometimes a layer of mortar or richer concrete with smaller aggregate is placed first to bond to the underlying lift. Lower strengths could be present for concrete where lift clean-up and material quality control was questionable.

Seepage on the downstream face of a concrete dam is not a reliable indicator of lift joint bond. Friant Dam has many large seeps on the downstream face along the lift lines but extracted core indicated bonded lift surfaces. It appeared that the bottom of the concrete lifts were not consolidated as well and were more porous than the overlying concrete, forming a seepage pathway, but that enough paste and contact was maintained to bond to the underlying lift. In contrast, Stewart Mountain Dam has a dry downstream face, but extracted core showed 16 of 23 lift joints unbonded. Failure to thoroughly clean the laitance from the lift surfaces resulted in weak bond, but there were sufficient fine particles at the interface to limit seepage along the joint. Therefore, it is important to locate as much information as possible about the methods and specific conditions encountered during construction, which are often keys to the strength of lift joints. If insufficient information can be located to make a judgment on lift joint strength at the time of the risk analysis, best practice is to perform some analysis results with poorly or unbonded lifts to judge the stability implications if this condition exists. It only takes one poorly bonded lift to create a potentially high risk situation.

It should be noted that any empirical relationships between concrete compressive strength and tensile strength do not apply to concrete that has been affected by alkali aggregate reaction (AAR). AAR results in formation of a gel around the aggregate particles. Therefore, while the compressive strength of the concrete may remain at a fairly high level, the tensile strength is often greatly reduced. Site specific testing is typically required in this case.

Cracked Base Analysis

The "cracked base" analysis has found its way into most concrete gravity dam design criteria, based on the "gravity" method of analysis, which assumes plane sections remain plane, and thus the distribution of vertical stress is linear. It is often applied without thoroughly evaluating the reasonableness of the results or the analysis assumptions relative to actual conditions. In a risk context, these must be considered. Several important points in this regard include:

- There is often confusion in how to deal with total stress and effective stress in carrying out the calculations. *Design of Small Dams* (1987) indicates that "Uplift from internal water pressures and stresses caused by the moment contribution from uplift along a horizontal plane are usually not included in the computation of σ_Z." This is the total stress method, which is endorsed by Watermeyer (2006), who states that the "reactive stress equations [which include the contribution from uplift] are erroneous and can lead to erroneous conclusions when uplift reducing drains are incorporated into the base of a gravity dam." That is not to say that uplift is not considered in the analysis, only that the moment contribution from internal uplift forces are not included in the stress calculations.
- The effective stress is determined by subtracting the pore water pressure (often equated to the "uplift pressure") from the total stress. If the effective stress is tensile and exceeds the tensile strength, then it is assumed that cracking can initiate. At that point, the water force in the crack becomes an "external" force which is included in the <u>total stress</u> calculations, and the base length is assumed to be shortened to only that portion downstream of the crack tip. The effective stress at the crack tip is subsequently calculated as the difference between the total stress and effective stress at that location. <u>It should be noted that the crack</u>

may not progress downstream of the point at which the effective stress is equal to the tensile strength.

- At the base of the dam, the potential for full reservoir pressure at the crack tip is controlled by the permeability of the foundation. Concrete gravity dams are typically founded on fractured and jointed rock. Thus, full reservoir pressure cannot develop at the tip of a crack along the foundation contact unless the foundation rock is massive and un-fractured, or the foundation joints are much tighter than the base crack. This is because water entering the crack will flow out through fractures at the base of the dam, and head loss will occur due to this flow. Thus, full uplift in a crack tip at the foundation contact may not be reasonable.
- Drains remain effective even if penetrated by a horizontal crack, although the drain efficiency may be reduced somewhat. This is demonstrated by the research sponsored by the Electric Power Research Institute at the University of Colorado (Amadei et al, 1991). Thus, analyses which consider full hydrostatic reservoir pressure in a crack tip downstream of the line of drains are typically not used for risk analyses.
- In the limiting case, if a crack is judged to propagate completely through the structure, the uplift pressure distribution along the crack is that which is appropriate for the post-cracking conditions, including the effects of drains in reducing the pressures, and pressures no higher than tailwater at the downstream face. It should be noted that there is very little guidance currently available concerning the effects of drains if the section cracks all the way through. If the aperture of the crack is thought to remain relatively tight in comparison to the drain diameter, the drains should retain some effectiveness. If the aperture is thought to be large in comparison to the drain diameter, then there may be more flow than the drains can handle, and their effectiveness would be questionable.
- If a crack is shown to exist, cohesion is presumed to act only on the portion of the intact potential sliding plane that is in compression. It is expected that intact concrete in tension will exhibit a smaller cohesive strength component, and since this is difficult to quantify, it is typically ignored.

Risks under Normal Operations

Concrete gravity dams that have performed well under normal operating conditions will likely continue to do so unless something changes. Changes could result from plugging of drains leading to an increase in uplift pressures, possible gradual creep that reduces the shear strength on potential sliding surfaces, or degradation of the concrete from alkali-aggregate reaction, freeze-thaw, or sulfate attack. These may be difficult to detect. A review of instrumentation results can be helpful. For example, if piezometers or uplift pressure gauges indicate a rise in pressures, and weirs indicate a reduction in drain flows, the drains may be plugging leading to potentially unstable conditions. If conditions appear to be changing, risk estimates are typically made for projected conditions as well as current conditions.

Reliability analysis for sliding on near horizontal foundation planes and/or potentially weak or cracked horizontal lift joints, typically using two-dimensional analysis sections, is the primary tool used for estimating risks posed by concrete gravity dams under normal operating conditions. This involves performing a probabilistic stability analysis using the Monte-Carlo technique as described in the section on Reliability Analysis. It requires an assessment of the likely range in input parameters, such as drain efficiency, cohesion and friction coefficient along the potential sliding surface, percentage of potential sliding surface that is intact, orientation of the potential sliding surface, and unit weight of the material(s). For potential foundation sliding planes, the influence of a downstream passive rock wedge should be considered, where appropriate. The shear strength of rough surfaces is nonlinear as a result of "riding up" over asperities at low normal stress and shearing through them at high normal stress. A straight line fit through such data points can result in overestimating the shear strength, particularly at low normal stresses. Therefore, strength parameters should be selected for the appropriate normal stress range of interest, or other means used to account for the nonlinear shear strength envelope.

Probabilistic stability analyses are typically performed at various reservoir water surface elevations, and combined in an event tree, such as that shown in Figure 20-1. See the sections on Event Trees and Reservoir Level Exceedance Curves for information on calculating reservoir load range probabilities. Note that in the limit, if small enough reservoir elevation increments are selected, a curve, referred to as a "fragility curve", results. The calculations are essentially the same whether larger discrete ranges or a fragility curve is used, and the results are similar as long as care is taken in selecting the discrete ranges. Therefore, either method can be used in estimating risks.

For the probabilistic stability analyses, it is important to examine the sensitivity rank coefficients and perform parametric studies, varying the parameters that affect the results the most. These parametric studies are used to estimate an appropriate range in conditional failure probabilities for the node titled "Sliding Instability". If there are significant three-dimensional effects, the two-dimensional sliding model may not be appropriate, and three-dimensional analyses may be needed to get a handle on how significant these effects might be if risks estimated from the two-dimensional models exceed the public protection guidelines.

Risks under Flood Loading

The approach for estimating risks due to structural instability under flood loading is essentially the same as for static loading, except that reservoir water surface elevations above the normal operating range, assigned the appropriate flood frequency, are used in the analyses and event tree. If flood routing information is not available, a conservative initial assumption is that inflow is equal to outflow, and the level of the reservoir is determined by that needed to pass a given peak inflow through the spillway and/or other release facilities (see also the section on Dam Overtopping). If the risks using this method are in an area where risk reduction actions are justified, then flood routings may be needed to get a better handle on the probability of attaining various reservoir elevations.

As the reservoir rises during flood loading, there may be a level at which the heel of the dam goes into tension (based on effective stress), in which case the potential for cracking along a lift joint at that elevation may increase. At some point, the estimated tensile strength of the concrete may be exceeded. Typically, a separation in the event tree reservoir load ranges occurs at these reservoir elevations. Stability analyses should be performed at these reservoir water elevations to judge the impact on the dam. Make sure the tailwater and uplift conditions correspond to the given reservoir elevation. In the case of an overflow section, care must be taken when assuming nappe forces (forces due to water flowing above the spillway) and tailwater forces act on the dam. Stilling basins can "sweep out" at high flows, and nappe pressures can become subatmospheric, reducing the stabilizing forces. Forces generated by water flowing through a flip bucket can also affect the results. A hydraulic evaluation is typically performed to determine whether it is appropriate to include these forces. A reliability model with the proper formulation for a cracked base analysis (see Watermeyer, 2006) is important in examining conditions where tension exceeding the tensile strength develops.

Risk evaluation associated with overtopping erosion of the abutments or foundation is discussed in the sections on Flood Overtopping and Erosion of Rock and Soil. However, another potentially significant issue involves cases where a concrete gravity dam serves as a spillway section. If erosion occurs at the downstream toe of the structure during spillway releases, weak bedding planes or foundation discontinuities in the underlying foundation rock might be exposed, daylighting into the erosion hole. This could remove passive resistance from the downstream rock mass, and result in a much more unstable condition. See the section on Erosion of Rock and Soil for guidance on how to estimate the potential for erosion. Figure 20-2 shows how this might impact the event tree. The potential for failure of stilling basins is discussed in the section on Overtopping of Walls and Stilling Basin Failure.

Risks under Earthquake Loading

Under earthquake loading, concrete gravity dams will respond according to the level and frequency of the shaking, and the reservoir level at the time of shaking. Therefore, sufficient analyses need to be performed to evaluate conditional failure probabilities at various levels of shaking and reservoir elevation. An example event tree to examine the potential for sliding failure through a weak lift line at a sharp change in slope on the downstream face is shown in Figure 20-3.

For each reservoir and seismic load range that is established for the estimating process, the likelihood of cracking through the dam body at this location must be estimated. The best approach for this is to perform a nonlinear dynamic finite element studies, modeling the potential weak plane with a contact surface that can be assigned a tensile strength value. As the tensile strength is exceeded near the faces during seismic response, the nodes will separate. If the shaking is severe enough, complete separation of the contact surface may propagate through the structure. Figure 20-4 shows a horizontal contact surface through a three-dimensional model of a concrete gravity dam. The darker color represents portions that remained un-cracked following the earthquake shaking. This indicates that at least one monolith cracked completely through for the set of assumptions used in this analysis. Similar studies can be performed using a two-dimensional section. By varying the tensile strength within reasonable parameters and monitoring the percentage of the joint that separates, a range in the likelihood of complete separation can be made. It should be noted that this is a total stress analysis, and pore pressures are not considered. Pore pressure behavior in concrete under dynamic loading is a subject of much uncertainty. Therefore, it is typically assumed that the total stress analysis provides a reasonable approximation of the potential for cracking through the section.

If the dam only cracks partially through, the probability of post-earthquake instability in the estimated cracked state is determined using static reliability analysis, as previously described. The estimated crack length from the nonlinear analysis of the seismic shaking is used as the starting point for a cracked base analysis. It is very difficult to estimate the amount and depth of cracking from a linear analysis. Linear analyses only help determine if and where cracks might initiate (high stress areas) but cannot model crack development or the sudden release of kinetic energy when cracks form.

If the section cracks all the way through, the likelihood of shearing the drains is next estimated. Information typically used to make this assessment includes calculated displacements from the finite element study assuming frictional resistance only on the potential sliding surface, as shown in Figure 20-5. In this case, very small values of damping, only enough to keep the model stable as the loading is applied, need to be used. If the model is over-dampened, the displacements will be under-estimated. Although this type of analysis assumes the section is cracked at the beginning of the earthquake and thus are somewhat conservative, they can be used to estimate the likelihood of drains, where present, being sheared. The post-earthquake instability could be considerably different whether the drains are still functioning after the earthquake shaking or not. It is possible that the drains could be sheared off, or opening of pathways in the foundation could lead to increased flow that overwhelms the drainage system. Therefore, two estimates are made, using reliability analysis, to account for these two conditions (drains functional or not), as indicated by the nodes on the event tree in Figure 20-3.

Seismic risk analysis of concrete gravity dams typically relies heavily on finite element analyses to evaluate the dynamic response, and the "gravity method" analyses to evaluate post-earthquake stability. The finite element analyses described above are not routinely performed. Although more uncertain, if analyses that include a contact surface are not available, it may be necessary to make judgments on cracking from traditional linear elastic finite element analysis results, by examining the magnitude and duration of the vertical tensile stresses at the upstream and downstream faces. Judgments must be made concerning how load is redistributed if cracking begins at the face, and how far toward the center of the dam it will progress, which is not an easy task. It is also important to examine the three-dimensional effects and, for example, whether excess driving load can be transferred to adjacent monoliths through shear keys. This is particularly true if all analyses are based on two-dimensional sections.

Lacking dynamic sliding analyses, a first approximation to the magnitude of displacement can be obtained from the following equations (Hendron, Cording, and Aiyer, 1980).

$$\delta = \frac{6V^2}{2gN} \qquad \text{for N/A} < 0.2$$

$$\delta = \left(\frac{V^2}{2gN}\right)\left(\frac{A}{N}\right) \qquad \text{for } 0.2 < \text{N/A} < 0.4$$

$$\delta = \left(\frac{V^2}{2gN}\right)\left(1 - \frac{N}{A}\right)\left(\frac{A}{N}\right) \qquad \text{for N/A} > 0.4$$

where g is the acceleration due to gravity, A is the peak ground acceleration as a fraction of gravity, V is the peak ground velocity, and N is the yield acceleration coefficient (expressed as a fraction of gravity and determined from a "gravity analysis" as the seismic coefficient that results in a factor of safety equal to 1.0 with all static loads applied to the structure). These equations are thought to be conservative in most cases. They were developed by Professor Newmark to delineate the upper bound of displacements for slopes from a large range in ground motions, consisting of soil records with significant low frequency content. Thus, longer pulses exceeding the yield acceleration were incorporated into their development than would be expected for rock records associated with gravity dams. However, the equations were developed from rigid-plastic analyses, and if there is significant structural response associated with a dam with respect to the applied ground motions, the displacements could possibly be larger.

Accounting for Uncertainty

Uncertainty is accounted for by estimating a range or distribution of values for each node on the event tree. A Monte-Carlo analysis is then run for the event tree to display the "cloud" of uncertainty, as described in the section on Combining and Portraying Risks. It is important to perform parametric or sensitivity analyses to examine how the results might change with different input parameters, especially for reliability analyses as described in the section on Probabilistic Stability Analysis. Different assumptions on the distribution and magnitude of water forces following an earthquake are typically made, since there is typically a great deal of uncertainty surrounding these values, and they can have a controlling effect on the results of the analyses. The uncertainty associated with how well the models are thought to actually predict the complex behavior should also be factored into the estimates, perhaps in a parametric sense (i.e. vary the corrections to account for model uncertainty and examine the results on the risk estimates).

Relevant Case Histories

Austin (Bayless) Dam: 1911

Austin Dam was a concrete gravity dam about 43 feet high and 534 feet long constructed by the Bayless Pulp and Paper Company about 1½ miles upstream of the town of Austin, Pennsylvania. A four-foot-thick by four-foot-deep concrete shear key was constructed into the horizontally bedded sandstone with interbedded weak shale layers. Anchor bars were grouted 5 to 8 feet into the foundation, extending well up into the dam body, on 2-foot 8-inch centers, located at about 6 feet from the upstream face. No drains were provided for the dam or foundation. During initial reservoir filling in 1910, the center portion of the dam at the overflow spillway section slid downstream about 18 inches at the base and 31 inches at the crest. The reservoir was lowered, but no repairs were made and the dam was put back into service. As the reservoir filled again, the dam suddenly gave way on September 30, 1911. More than 75 people lost their lives in Austin. Back analysis suggests that sliding occurred on a weak shale layer within the foundation (Anderson et al, 1998).

Bouzey Dam: 1895

Bouzey Dam was a 72-foot high masonry gravity dam constructed across the L'Aviere River near Epinal, France. Similar to Austin Dam, the dam was founded on horizontally interbedded sandstone and lenticular clay seams, with no drainage provisions, and about a 6-foot wide by 10-foot deep cutoff key constructed into the rock at the upstream face of the dam. Also similar to Austin Dam, an incident occurred during initial filling whereby the center section of the dam moved downstream about a foot, shearing the key. Unlike Austin Dam, the reservoir was lowered and the lower portion of the dam was strengthened. Unfortunately, the upper portion of the dam was quite thin (less than 18 feet thick for about the upper 35 feet), and upon refilling, the dam cracked and the upper 30 feet or so was sheared off and swept away. Stability calculations indicate that cracking was likely at the elevation where the shear failure occurred, and once cracked through, the upper portion of the dam was unstable. (Anderson et al, 1998).

Koyna Dam: 1967

Koyna Dam is a 338-foot-high and 2,800-foot long concrete gravity dam constructed on the Koyna River in southwestern India between 1954 and 1963. During construction the decision was made to raise the dam and the downstream slope of the non-overflow section was steepened in the upper 120 feet of the structure to accommodate the raise, resulting in a discontinuous change in slope at that location. The dam was shaken by a M6.5 earthquake on December 11, 1967. A strong motion accelerograph located in a gallery on the upper right abutment recorded a peak ground acceleration of 0.63g cross-canyon, 0.49g downstream, and 0.34g vertical. Although the dam did not fail, deep horizontal cracks formed throughout the upstream and downstream faces near the change in slope where a stress concentration is expected to occur, requiring the installation of tendons and construction of buttresses on the downstream face to stabilize the structure. Finite element analyses indicated stress concentrations near the change in slope that exceed the dynamic tensile strength of the concrete (Anderson et al, 1998).

Exercise

Given: The upper 34.4 feet of a concrete gravity dam, above a lift joint, with base thickness of 16.2 feet and a reservoir loading of 32.6 feet above the base; Calculate the total stress and effective stress at the upstream face in this location. The weight of this section of the dam is 64.4 kips/ft, and the moment induced by the reservoir load on the upstream face and the dam weight together is 279 kip-ft/ft (downstream rotation). (The moment of inertia is equal to the base thickness cubed divided by 12.) Is the dam likely to crack at this location if it is constructed of conventional concrete with an unconfined compressive strength of 3,500 lb/in^2 and 6-inch maximum size aggregate?

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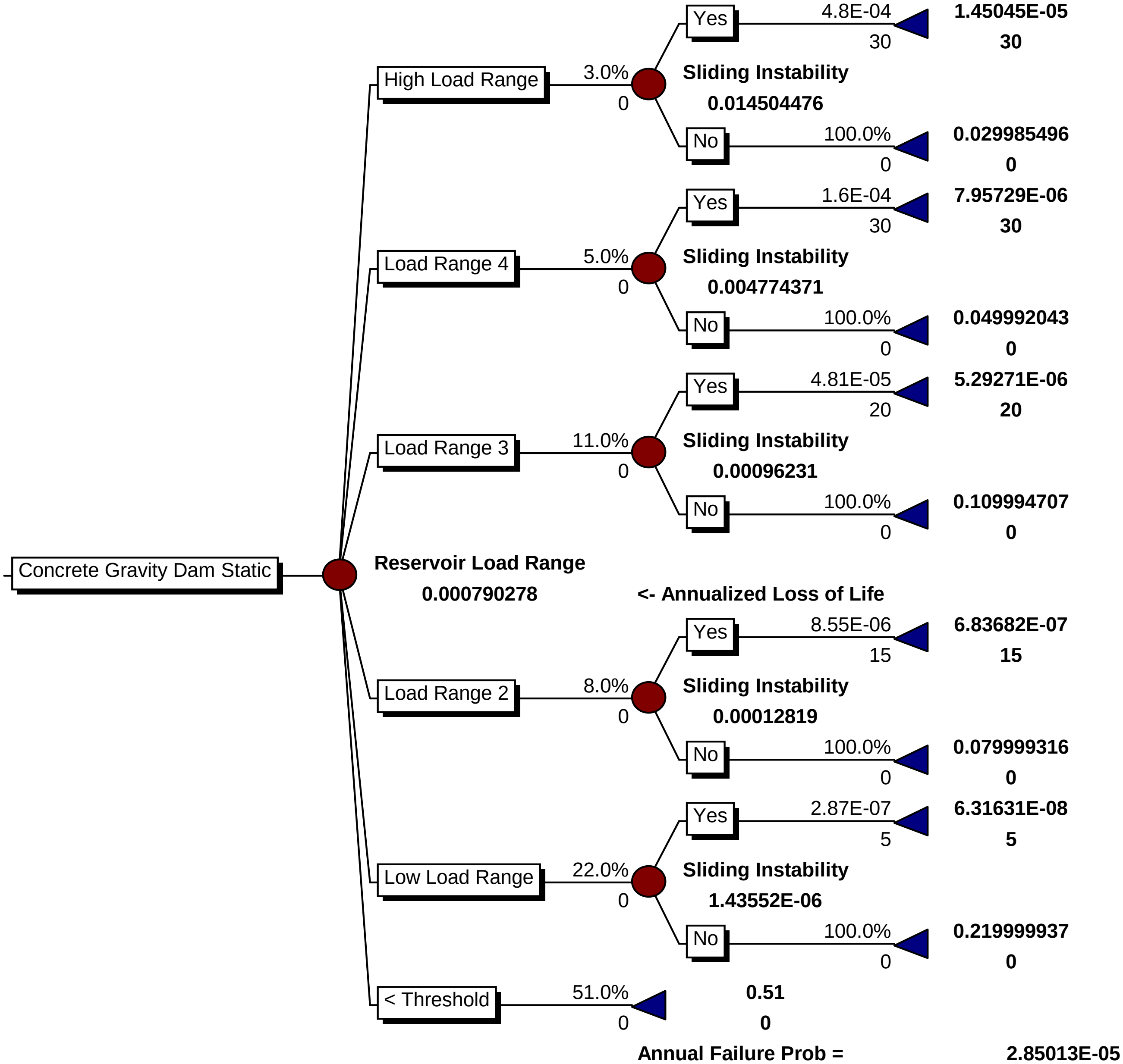

Figure 20-1. Concrete Gravity Dam Instability, Static Load Event Tree

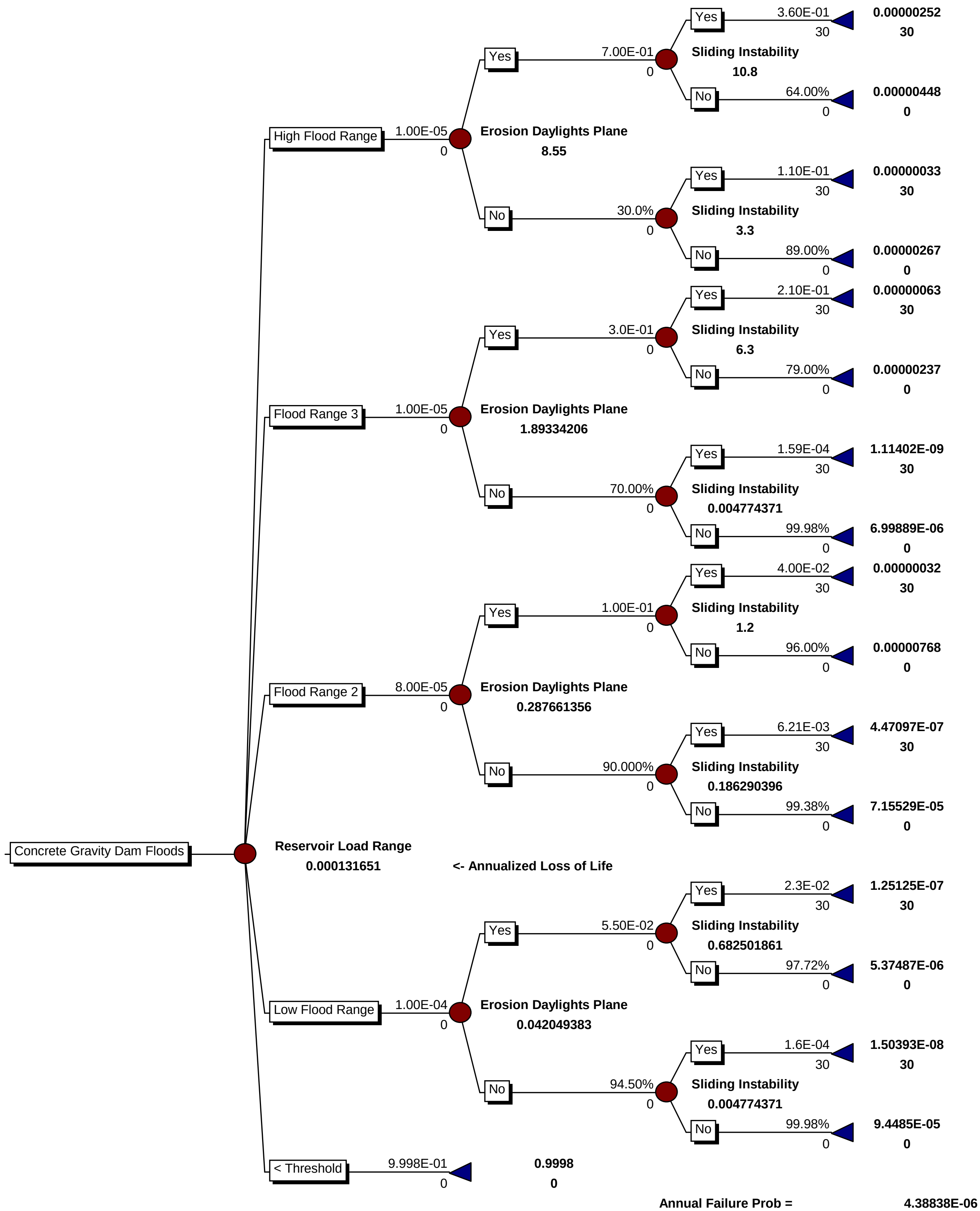

Figure 20-2. Concrete Gravity Dam Instability, Flood Loading Event Tree

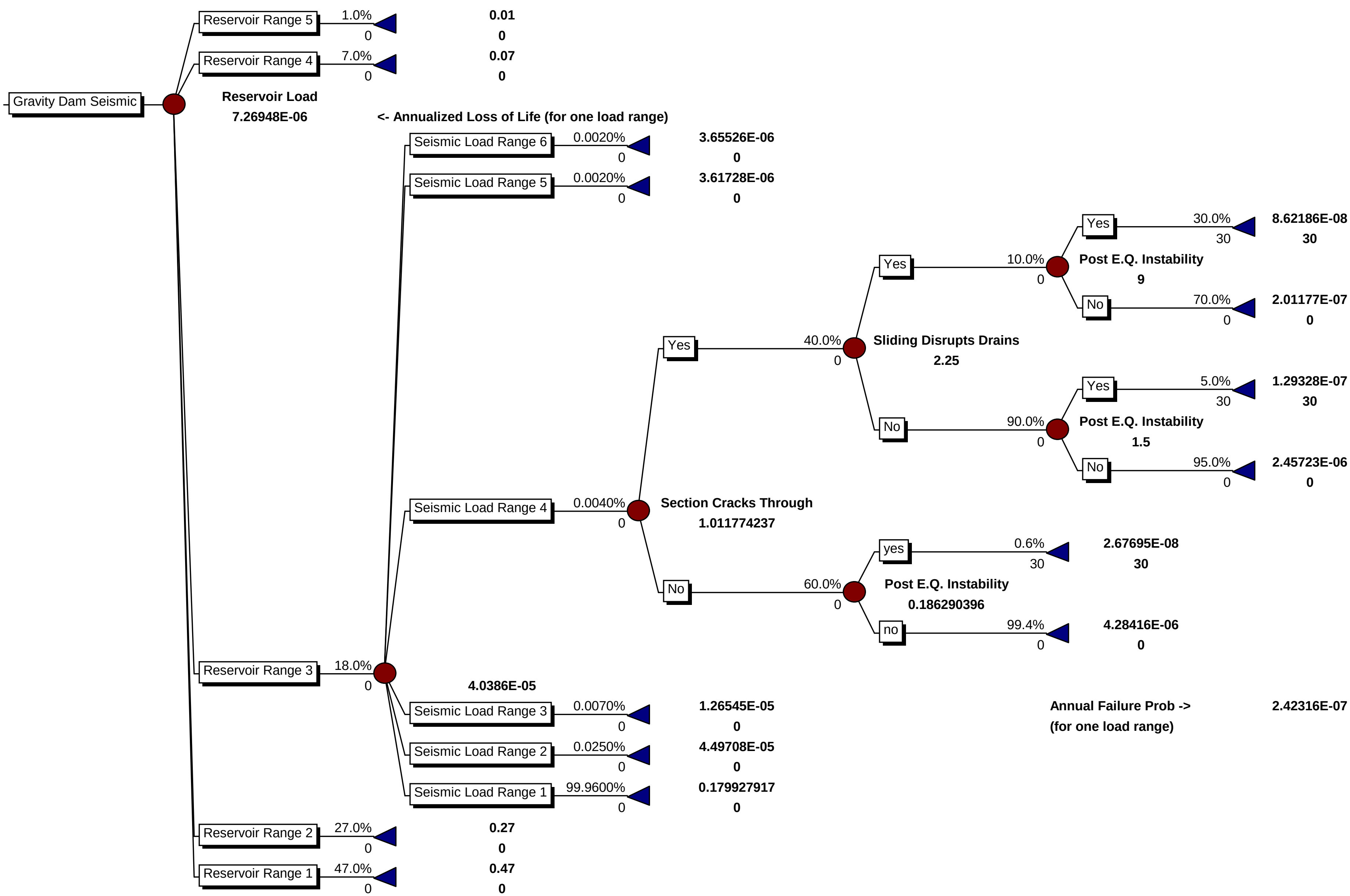

Figure 20-3. Concrete Gravity Dam Instability, Seismic Loading

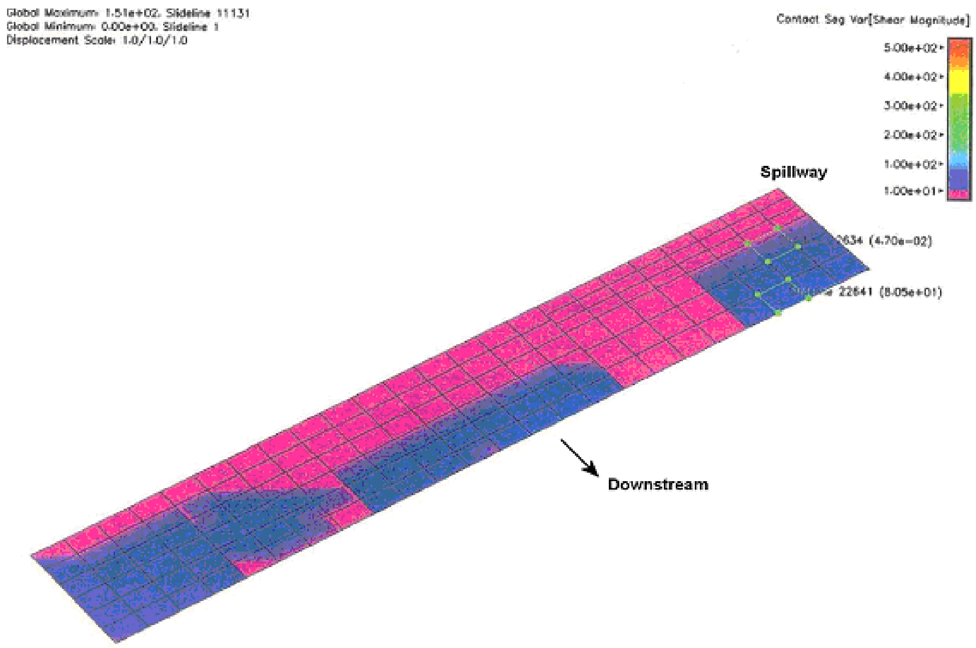

Figure 20-4. Separation of Contact Surface in Dynamic Finite Element Analysis (lighter color indicates separation)

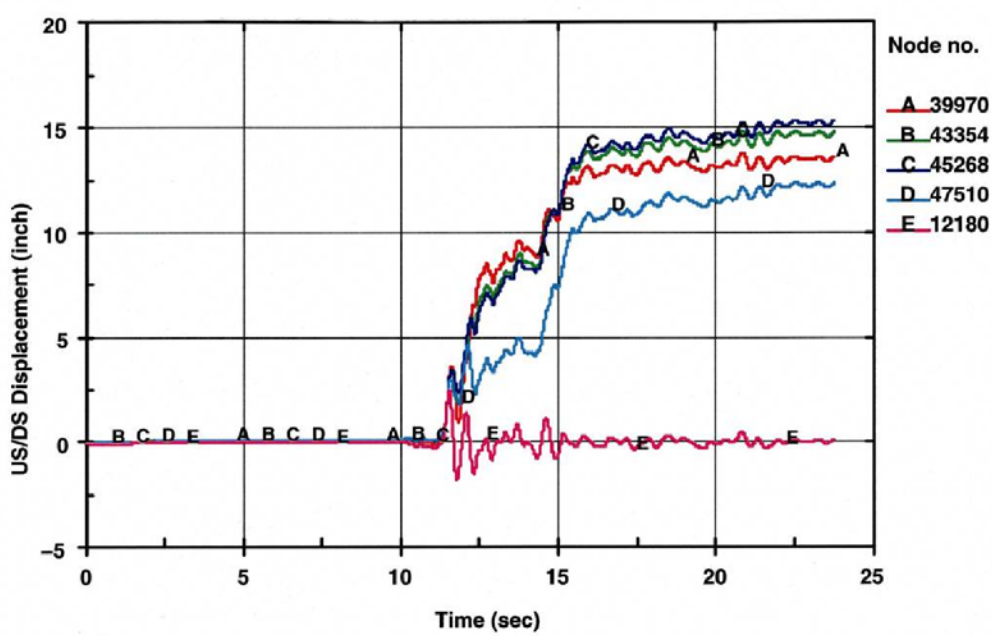

Figure 20-5. Displacement of Various Monoliths during Dynamic Loading (friction only, curve E is at base of sliding contact surface)

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